

# OUTLET STRUCTURES

## Introduction

The outfall of stormwater runoff storage facilities directly impacts down stream properties and receiving waters. The location of the outfall structure is therefore crucial in the design of an overall stormwater management system. The outfall from a storage facility should be located to access the nearest natural drainage feature or stormwater conveyance system with adequate capacity to convey released flow from the storage facility. Overflow or emergency spillways designed to accommodate a 100-year frequency storm may not be located near the principal outlet structure. The spillway shall not be located in fill areas. The spillway shall not be designed to discharge towards structures.

The following information shall be provided for evaluation of the stormwater release locations:

1. Indicate and label the drainage feature located immediately downstream of the outlet structure.
2. Indicate the nearest receiving creek, stream or tributary as delineated by FEMA or USGS quadrangle maps and distance to such feature.
3. Indicate outlet location and discharge flows on plans.
4. Provide energy dissipation structures in addition to rip-rap to control outlet velocities for erosion control, as necessary.

## 2.3.1 Symbols and Definitions

To provide consistency within this section as well as throughout this Manual, the symbols listed in Table 2.3.1-1 will be used. These symbols were selected because of their wide use in technical publications. In some cases, the same symbol is used in existing publications for more than one definition. Where this occurs in this section, the symbol will be defined where it occurs in the text or equations.

**Table 2.3.1-1** Symbols and Definitions

Symbol	Definition	Units
A, a	Cross sectional or surface area	ft <sup>2</sup>
A <sub>m</sub>	Drainage area	mi <sup>2</sup>
B	Breadth of weir	ft
C	Weir coefficient	-
d	Change in elevation	ft
D	Depth of basin or diameter of pipe	ft
g	Acceleration due to gravity	ft / s <sup>2</sup>
H	Head on structure	ft
H <sub>c</sub>	Height of weir crest above channel bottom	ft
K, k	Coefficient	-
L	Length	ft
n	Manning's n	-
Q, q	Peak inflow or outflow rate	cfs, in
V <sub>u</sub>	Approach velocity	ft / s
WQ <sub>v</sub>	Water quality volume	ac ft
w	Maximum cross sectional bar width facing the flow	in
x	Minimum clear spacing between bars	in
θ	Angle of v-notch	degrees
θ <sub>g</sub>	Angle of the grate with respect to the horizontal	degrees

## 2.3.2 Primary Outlets

### 2.3.2.1 Introduction

Primary outlets provide the critical function of the regulation of flow for structural stormwater controls. There are several different types of outlets that may consist of a single stage outlet structure, or several outlet structures combined to provide multi-stage outlet control.

For a single stage system, the stormwater facility can be designed as a simple pipe or culvert. For multistage control structures, the inlet is designed considering a range of design flows.

A stage-discharge curve is developed for the full range of flows that the structure would experience. The outlets are housed in a riser structure connected to a single outlet conduit. An alternative approach would be to provide several pipe or culvert outlets at different levels in the basin that are either discharged separately or are combined to discharge at a single location.

This section provides an overview of outlet structure hydraulics and design for stormwater storage facilities. The design engineer is referred to an appropriate hydraulics text for additional information on outlet structures not contained in this section.

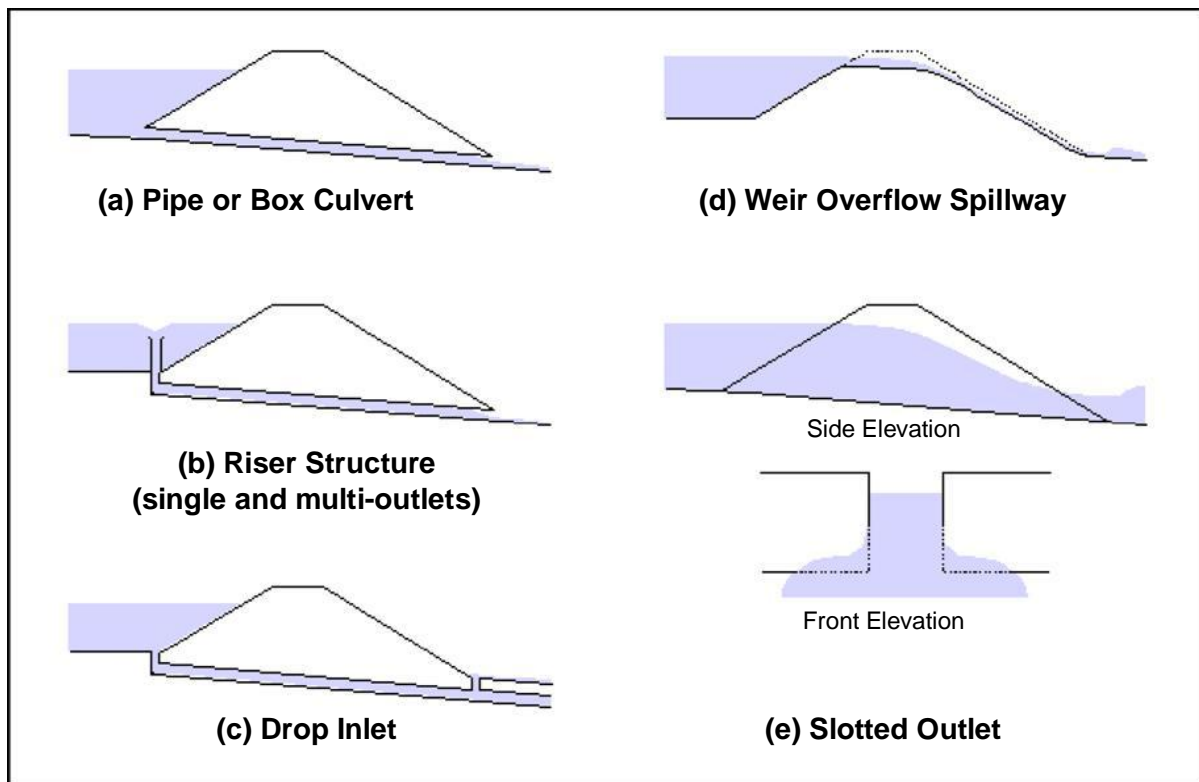


Figure 2.3.2-1 Typical Primary Outlets

### 2.3.2.2 Outlet Structure Types

There are a wide variety of outlet structure types, the most common of which are covered in this section. Descriptions and equations are provided for the following outlet types for use in stormwater facility design:

- Orifices
- V-notch weirs
- Pipes / Culverts
- Sharp-crested weirs
- Broad-crested weirs
- Proportional weirs
- Combination outlets

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Each of these outlet types has a different design purpose and application:

- Water quality and channel protection flows are normally handled with smaller, more protected outlet structures such as reverse slope pipes, hooded orifices, orifices located within screened pipes or risers, perforated plates or risers, and V-notch weirs.
- Larger flows, such as overbank protection and extreme flood flows, are typically handled through a riser with different sized openings, through an overflow at the top of a riser (drop inlet structure), or a flow over a broad crested weir or spillway through the embankment. Overflow weirs can also be of different heights and configurations to handle control of multiple design flows.

### 2.3.2.3 Orifices

An orifice is a circular or rectangular opening of a prescribed shape and size. The flow rate depends on the height of the water above the opening and the size and edge treatment of the orifice.

For a single orifice, as illustrated in Figure 2.3.2-2(a), the orifice discharge can be determined using the standard orifice equation below.

$$Q = CA(2gH)^{0.5} \quad (2.3.1)$$

Where: Q = the orifice flow discharge (cfs)

C = discharge coefficient

A = cross-sectional area of orifice or pipe (ft<sup>2</sup>)

g = acceleration due to gravity (32.2 ft/s<sup>2</sup>)

D = diameter of orifice or pipe (ft)

H = effective head on the orifice, from the center of orifice to the water surface

If the orifice discharges as a free outfall, then the effective head is measured from the center of the orifice to the upstream (headwater) surface elevation. If the orifice discharge is submerged, then the effective head is the difference in elevation of the headwater and tailwater surfaces as shown in Figure 2.3.2-2(b).

When the material is thinner than the orifice diameter, with sharp edges, a coefficient of 0.6 should be used. For square-edged entrance conditions the generic orifice equation can be simplified:

$$Q = 0.6A(2gH)^{0.5} = 3.78D^2H^{0.5} \quad (2.3.2)$$

When the material is thicker than the orifice diameter a coefficient of 0.80 should be used. If the edges are rounded, a coefficient of 0.92 can be used.

Flow through multiple orifices, such as the perforated plate shown in Figure 2.3.2-2(c), can be computed by summing the flow through individual orifices. For multiple orifices of the same size and under the influence of the same effective head, the total flow can be determined by multiplying the discharge for a single orifice by the number of openings.

Perforated orifice plates for the control of discharge can be of any size and configuration. However, the Denver Urban Drainage and Flood Control District has developed standardized dimensions that have worked well. Table 2.3.2-1 gives appropriate dimensions. The vertical spacing between hole centerlines is always 4 inches.

<b>Table 2.3.2-1 Circular Perforation Sizing</b>				
<b>Hole Diameter (in)</b>	<b>Minimum Column Hole Centerline Spacing (in)</b>	<b>Flow Area per Row (in<sup>2</sup>)</b>		
		<b>1 column</b>	<b>2 columns</b>	<b>3 columns</b>
1/4	1	0.05	0.10	0.15
5/16	2	0.08	0.15	0.23
3/8	2	0.11	0.22	0.33
7/16	2	0.15	0.30	0.45
1/2	2	0.20	0.40	0.60
9/16	3	0.25	0.50	0.75
5/8	3	0.31	0.62	0.93
11/16	3	0.37	0.74	1.11
3/4	3	0.44	0.88	1.32
13/16	3	0.52	1.04	1.56
7/8	3	0.60	1.20	1.80
15/16	3	0.69	1.38	2.07
1	4	0.79	1.58	2.37
1 1/16	4	0.89	1.78	2.67
1 1/8	4	0.99	1.98	2.97
1 3/16	4	1.11	2.22	3.33
1 1/4	4	1.23	2.46	3.69
1 5/16	4	1.35	2.70	4.05
1 3/8	4	1.48	2.96	4.44
1 7/16	4	1.62	3.24	4.86
1 1/2	4	1.77	3.54	5.31
1 9/16	4	1.92	3.84	5.76
1 5/8	4	2.07	4.14	6.21
1 11/16	4	2.24	4.48	6.72
1 3/4	4	2.41	4.82	7.23
1 13/16	4	2.58	5.16	7.74
1 7/8	4	2.76	5.52	8.28
1 15/16	4	2.95	5.90	8.85
2	4	3.14	6.28	9.42
Number of columns refers to parallel columns of holes				
Minimum steel plate thickness		1/4"	5/16"	3/8"
Source: Urban Drainage and Flood Control District, Denver, CO				

For rectangular slots the height is normally 2 inches with variable width. Only one column of rectangular slots is allowed.

Figure 2.3.2-3 provides a schematic of an orifice plate outlet structure for a wet ED pond showing the design pool elevations and the flow control mechanisms.

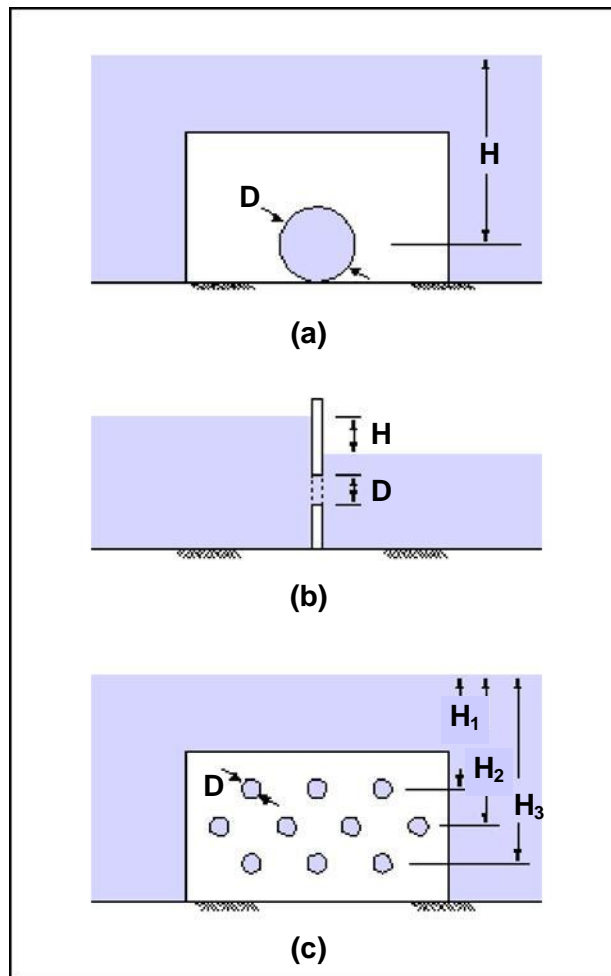


Figure 2.3.2-2 Orifice Definitions

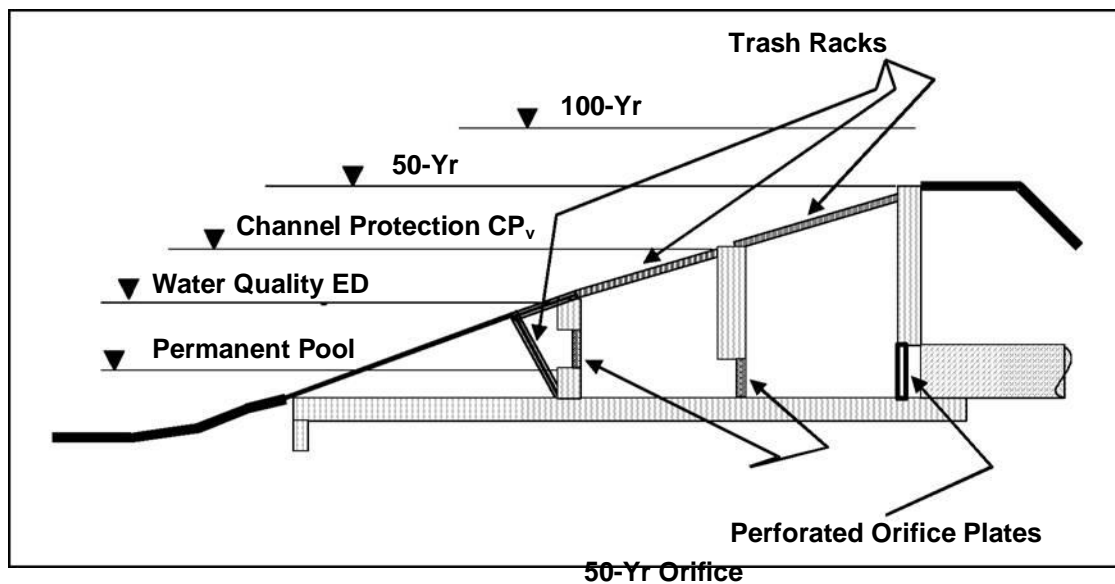


Figure 2.3.2-3 Schematic of Orifice Plate Outlet Structure

### 2.3.2.4 Pipes and Culverts

Discharge pipes are often used as outlet structures for stormwater control facilities. The design of these pipes can be for either single or multi-stage discharges. A reverse-slope underwater pipe is often used for water quality or channel protection outlets.

Pipes which are smaller than 12" in diameter may be analyzed as a submerged orifice as long as  $H/D$  is greater than 1.5. *Note: For low flow conditions when the flow reaches and begins to overflow the pipe, weir flow controls (see subsection 2.3.2.5). As the stage increases the flow will transition to orifice flow.*

Pipes greater than 12 inches in diameter should be analyzed as a discharge pipe with headwater and tailwater effects taken into account. The outlet hydraulics for pipe flow can be determined from the outlet control culvert nomographs and procedures given in Section 4.3, *Culvert Design*, or by using equation 2.3.4 (NRCS, 1984).

The following equation is a general pipe flow equation that is derived through the use of the Bernoulli and continuity principles.

$$Q = a[(2gH)/(1 + k_m + k_p L)]^{0.5} \quad (2.3.4)$$

Where:  $Q$  = discharge (cfs)  
 $a$  = pipe cross sectional area (ft<sup>2</sup>)  
 $g$  = acceleration of gravity (ft/s<sup>2</sup>)  
 $H$  = elevation head differential (ft)  
 $k_m$  = coefficient of minor losses (use 1.0)  
 $k_p$  = pipe friction coefficient =  $5087n^2/D^{4/3}$   
 $L$  = pipe length (ft)

### 2.3.2.5 Sharp-Crested Weirs

If the overflow portion of a weir has a sharp, thin leading edge such that the water springs clear as it overflows, the overflow is termed a *sharp-crested weir*. If the sides of the weir also cause the through flow to contract, it is termed an *end-contracted* sharp-crested weir. Sharp-crested weirs have stable stage-discharge relations and are often used as a measurement device. A sharp-crested weir with no end contractions is illustrated in Figure 2.3.2-4(a). The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.27 + 0.4(H/H_c))] LH^{1.5} \quad (2.3.5)$$

Where:  $Q$  = discharge (cfs)  
 $H$  = head above weir crest excluding velocity head (ft)  
 $H_c$  = height of weir crest above channel bottom (ft)  
 $L$  = horizontal weir length (ft)

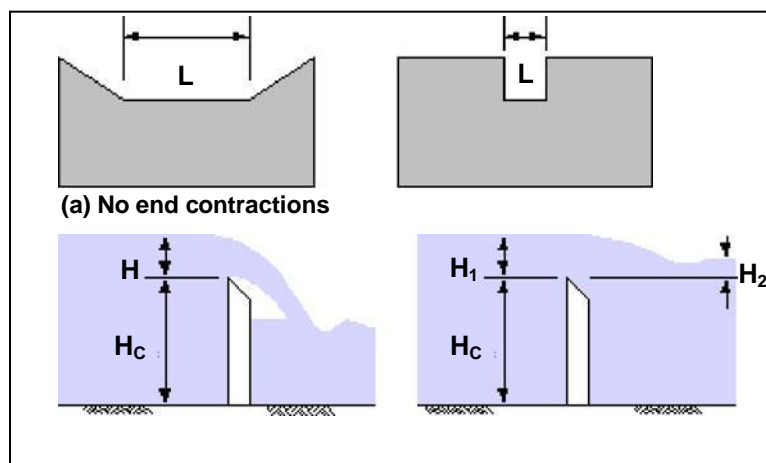


Figure 2.3.2-5 Sharp-Crested Weir  
(b) With end contractions  
(c) Section View

A sharp-crested weir with two end contractions is illustrated in Figure 2.3.2-4(b). The discharge equation for this configuration is (Chow, 1959):

$$Q = [(3.24 + 0.04(H/H_C)) (L - 0.2H) H^{1.5} \quad (2.3.6)$$

Where:  $Q$  = discharge (cfs)  
 $H$  = head above weir crest excluding velocity head (ft)  
 $H_C$  = height of weir crest above channel bottom (ft)  
 $L$  = horizontal weir length (ft)

A sharp-crested weir will be affected by submergence when the tailwater rises above the weir crest elevation. The result will be that the discharge over the weir will be reduced. The discharge equation for a sharp-crested submerged weir is (Brater and King, 1976):

$$Q_S = Q_f (1 - (H_2 / H_1)^{1.5})^{0.385} \quad (2.3.7)$$

Where:  $Q_S$  = submergence flow (cfs)  
 $Q_f$  = free flow (cfs)  
 $H_1$  = upstream head above crest (ft)  
 $H_2$  = downstream head above crest (ft)

### 2.3.2.6 Broad-Crested Weirs

A weir in the form of a relatively long raised channel control crest section is a *broad-crested* weir. The flow control section can have different shapes, such as triangular or circular. True broad-crested weir flow occurs when upstream head above the crest is between the limits of about 1/20 and 1/2 the crest length in the direction of flow. For example, a thick wall or a flat stop log can act like a sharp-crested weir when the approach head is large enough that the flow springs from the upstream corner. If upstream head is small enough relative to the top profile length, the stop log can act like a broad-crested weir (USBR, 1997).

The equation for the broad-crested weir is (Brater and King, 1976):

$$Q = CLH^{1.5} \quad (2.3.8)$$

Where:  $Q$  = discharge (cfs)  
 $C$  = broad-crested weir coefficient  
 $L$  = broad-crested weir length perpendicular to flow (ft)  
 $H$  = head above weir crest (ft)

If the upstream edge of a broad-crested weir is so rounded as to prevent contraction and if the slope of the crest is as great as the loss of head due to friction, flow will pass through critical depth at the weir crest; this gives the maximum  $C$  value of 3.087. For sharp corners on the broad-crested weir, a minimum  $C$  value of 2.6 should be used. Information on  $C$  values as a function of weir crest breadth and head is given in Table 2.3.2-2.

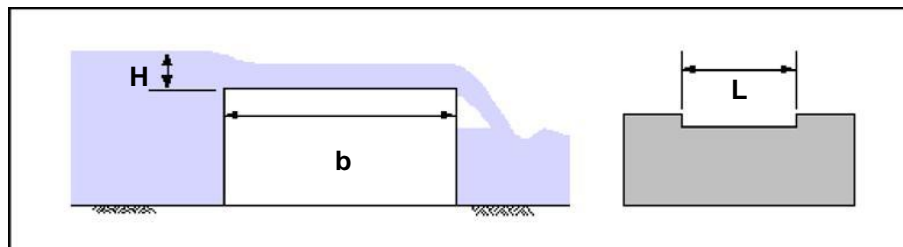


Figure 2.3.2-6 Broad-Crested Weir

Table 2.3.2-2 Broad-Crested Weir Coefficient (C) Values											
Measured Head (H)* In feet	Weir Crest Breadth (b) in feet										
	0.50	0.75	1.00	1.50	2.00	2.50	3.00	4.00	5.00	10.00	15.00
0.2	2.80	2.75	2.69	2.62	2.54	2.48	2.44	2.38	2.34	2.49	2.68
0.4	2.92	2.80	2.72	2.64	2.61	2.60	2.58	2.54	2.50	2.56	2.70
0.6	3.08	2.89	2.75	2.64	2.61	2.60	2.68	2.69	2.70	2.70	2.70
0.8	3.30	3.04	2.85	2.68	2.60	2.60	2.67	2.68	2.68	2.69	2.64
1.0	3.32	3.14	2.98	2.75	2.66	2.64	2.65	2.67	2.68	2.68	2.63
1.2	3.32	3.20	3.08	2.86	2.70	2.65	2.64	2.67	2.66	2.69	2.64
1.4	3.32	3.26	3.20	2.92	2.77	2.68	2.64	2.65	2.65	2.67	2.64
1.6	3.32	3.29	3.28	3.07	2.89	2.75	2.68	2.66	2.65	2.64	2.63
1.8	3.32	3.32	3.31	3.07	2.88	2.74	2.68	2.66	2.65	2.64	2.63
2.0	3.32	3.31	3.30	3.03	2.85	2.76	2.27	2.68	2.65	2.64	2.63
2.5	3.32	3.32	3.31	3.28	3.07	2.89	2.81	2.72	2.67	2.64	2.63
3.0	3.32	3.32	3.32	3.32	3.20	3.05	2.92	2.73	2.66	2.64	2.63
3.5	3.32	3.32	3.32	3.32	3.32	3.19	2.97	2.76	2.68	2.64	2.63
4.0	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.70	2.64	2.63
4.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.74	2.64	2.63
5.0	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.07	2.79	2.64	2.63
5.5	3.32	3.32	3.32	3.32	3.32	3.32	3.32	3.32	2.88	2.64	2.63
* Measured at least 2.5H upstream of the weir. Source: Brater and King (1976)											

### 2.3.2.7 V-Notch Weirs

The discharge through a V-notch weir (Figure 2.3.2-6) can be calculated from the following equation (Brater and King, 1976).

$$Q = 2.5 \tan (\theta / 2) H^{2.5} \quad (2.3.9)$$

Where: Q = discharge (cfs)  
 $\theta$  = angle of V-notch (degrees)  
H = head on apex of notch (ft)

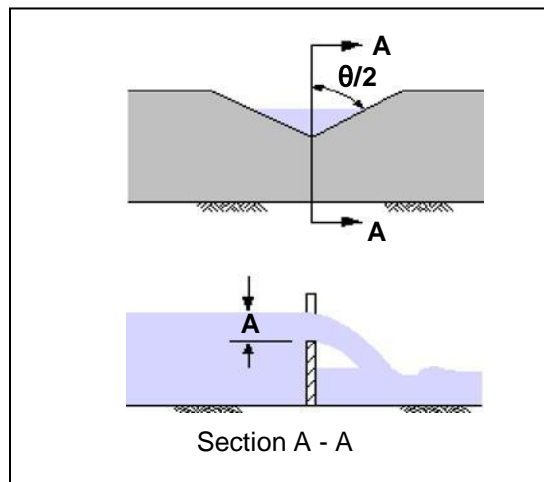


Figure 2.3.2-7 V-Notch Weir



### 2.3.2.8 Proportional Weirs

Although more complex to design and construct, a proportional weir may significantly reduce the required storage volume for a given site. The proportional weir is distinguished from other control devices by having a linear head-discharge relationship achieved by allowing the discharge area to vary nonlinearly with head. A typical proportional weir is shown in Figure 2.3.2-7. Design equations for proportional weirs are (Sandvik, 1985):

$$Q = 4.97 a^{0.5} b (H - a/3) \quad (2.3.10)$$

$$x/b = 1 - (1/3.17)(\arctan(y/a)^{0.5}) \quad (2.3.11)$$

Where:  $Q$  = discharge (cfs)

Dimensions  $a$ ,  $b$ ,  $H$ ,  $x$ , and  $y$  are shown in Figure 2.3.2-8

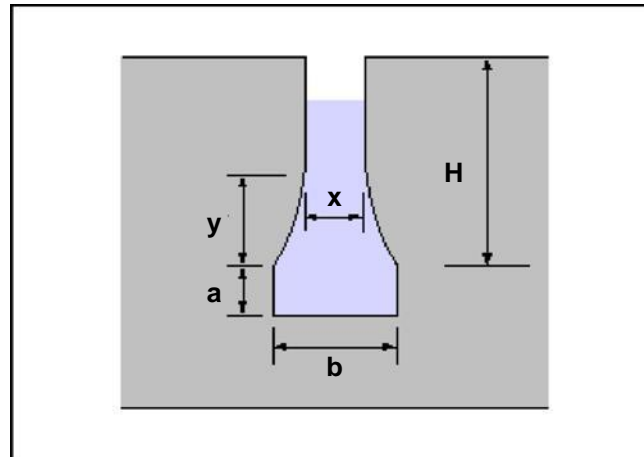


Figure 2.3.2-8 Proportional Weir Dimensions

### 2.3.2.9 Combination Outlets

Combinations of orifices, weirs and pipes can be used to provide multi-stage outlet control for different control volumes within a storage facility (i.e., water quality volume, channel protection volume, overbank flood protection volume, and/or extreme flood protection volume).

They are generally of two types of combination outlets: shared outlet control structures and separate outlet controls. Shared outlet control is typically a number of individual outlet openings (orifices), weirs or drops at different elevations on a riser pipe or box which all flow to a common larger conduit or pipe. Figure 2.3.2-9 shows an example of a riser designed for a wet ED pond. The orifice plate outlet structure in Figure 2.3.2-3 is another example of a combination outlet.

Separate outlet controls are less common and may consist of several pipe or culvert outlets at different levels in the storage facility that are either discharged separately or are combined to discharge at a single location.

The use of a combination outlet requires the construction of a composite stage-discharge curve (as shown in Figure 2.3.2-9) suitable for control of multiple storm flows. The design of multistage combination outlets is discussed later in this section.

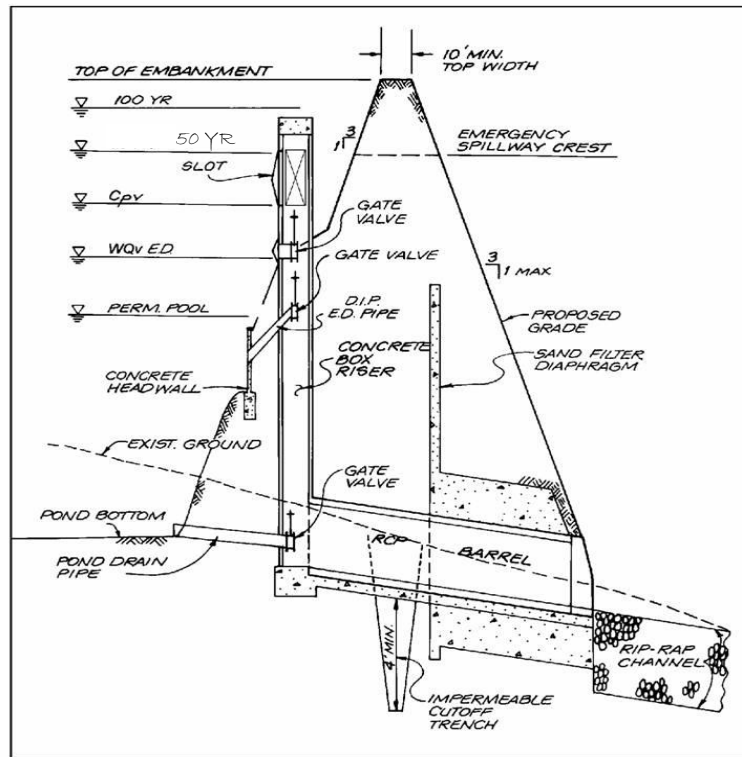


Figure 2.3.2-9 Schematic of Combination Outlet Structure

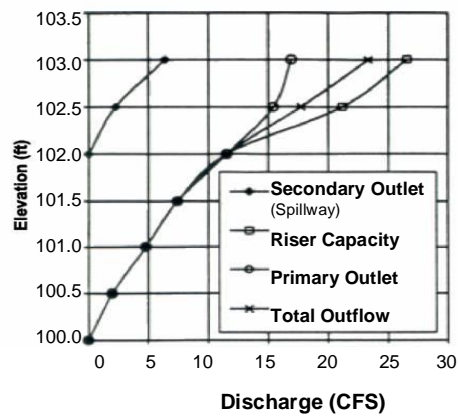


Figure 2.3.2-10 Composite Stage-Discharge Curve

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## 2.3.3 Extended Detention (Water Quality and Channel Protection) Outlet Design

### 2.3.3.1 Introduction

Extended detention orifice sizing is required in design applications that provide extended detention for downstream channel protection or the ED portion of the water quality volume. In both cases an extended detention orifice or reverse slope pipe can be used for the outlet. For a structural control facility providing both  $WQ_v$  extended detention and  $CP_v$  control (wet ED pond, micro-pool ED pond, and shallow ED wetland), there will be a need to design two outlet orifices – one for the water quality control outlet and one for the channel protection drawdown.

*(This following procedures are based on the water quality outlet design procedures included in the Virginia Stormwater Management Handbook, 1999)*

The outlet hydraulics for peak control design (overbank flood protection and extreme flood protection) is usually straightforward in that an outlet is selected that will limit the peak flow to some predetermined maximum. Since volume and the time required for water to exit the storage facility are not usually considered, the outlet design can easily be calculated and routing procedures used to determine if quantity design criteria are met.

In an extended detention facility for water quality treatment or downstream channel protection, however, the storage volume is detained and released over a specified amount of time (e.g., 24-hours). The release period is a brim drawdown time, beginning at the time of peak storage of the water quality volume until the entire calculated volume drains out of the basin. This assumes that the brim volume is present in the basin prior to any discharge. In reality, however, water is flowing out of the basin prior to the full or brim volume being reached. Therefore, the extended detention outlet can be sized using either of the following methods:

- (1) Use the maximum hydraulic head associated with the storage volume and maximum flow, and calculate the orifice size needed to achieve the required drawdown time, and route the volume through the basin to verify the actual storage volume used and the drawdown time.
- (2) Approximate the orifice size using the average hydraulic head associated with the storage volume and the required drawdown time.

These two procedures are outlined in the examples below and can be used to size an extended detention orifice for water quality and/or channel protection.

### 2.3.3.2 Method 1: Maximum Hydraulic Head with Routing

A wet ED pond sized for the required water quality volume will be used here to illustrate the sizing procedure for an extended-detention orifice.

Given the following information, calculate the required orifice size for water quality design.

Given: Water Quality Volume ( $WQ_v$ ) = **0.76 ac ft = 33,106 ft<sup>3</sup>**

Maximum Hydraulic Head ( $H_{max}$ ) = 5.0 ft (from stage vs. storage data)

**Step 1:** Determine the maximum discharge resulting from the 24-hour drawdown requirement. It is calculated by dividing the Water Quality Volume (or Channel Protection Volume) by the required time to find the average discharge, and then multiplying by two to obtain the maximum discharge.

$$Q_{avg} = 33,106 \text{ ft}^3 / (24 \text{ hr})(3,600 \text{ s/hr}) = 0.38 \text{ cfs}$$

$$Q_{max} = 2 \times Q_{avg} = 2 \times 0.38 = 0.76 \text{ cfs}$$

**Step 2:** Determine the required orifice diameter by using the orifice equation (2.3.1) and  $Q_{max}$  and  $H_{max}$ :

$$Q = CA (2gH)^{0.5}, \text{ or } A = Q / C (2gH)^{0.5}$$

$$A = 0.76 / 0.6[(2)(32.2)(5.0)]^{0.5} = 0.071 \text{ ft}^3$$

Determine pipe diameter from  $A = 3.14d^2 / 4$ , then  $d = (4A/3.14)^{0.5}$

$$D = [4(0.071) / 3.14]^{0.5} = 0.30 \text{ ft} = 3.61 \text{ in}$$

*Use a 3.6-inch diameter water quality orifice.*

Routing the water quality volume of 0.76 ac ft through the 3.6-inch water quality orifice will allow the designer to verify the drawdown time, as well as the maximum hydraulic head elevation. The routing effect will result in the actual drawdown time being less than the calculated 24 hours. Judgment should be used to determine whether the orifice size should be reduced to achieve the required 24 hours or if the actual time achieved will provide adequate pollutant removal.

### 2.3.3.3 Method 2: Average Hydraulic Head and Average Discharge

Using the data from the previous example (2.3.3.2), use Method 2 to calculate the size of the outlet orifice.

Given: Water Quality Volume ( $WQ_v$ ) = **0.76 ac ft = 33,106 ft<sup>3</sup>**

Average Hydraulic Head ( $h_{avg}$ ) = 2.5 ft (from stage vs. storage data)

**Step 1:** Determine the average release rate to release the water quality volume over a 24-hour time period.

$$Q_{avg} = 33,106 \text{ ft}^3 / (24 \text{ hr})(3,600 \text{ s/hr}) = 0.38 \text{ cfs}$$

**Step 2:** Determine the required orifice diameter by using the orifice equation (2.3.1) and the average head on the orifice:

$$Q = CA (2gH)^{0.5}, \text{ or } A = Q / C (2gH)^{0.5}$$

$$A = 0.38 / 0.6[(2)(32.2)(2.5)]^{0.5} = 0.05 \text{ ft}^3$$

Determine pipe diameter from  $A = 3.14r^2 = 3.14d^2 / 4$ , then  $d = (4A/3.14)^{0.5}$

$$D = [4(0.05) / 3.14]^{0.5} = 0.252 \text{ ft} = 3.03 \text{ in}$$

*Use a 3-inch diameter water quality orifice.*

Use of Method 1, utilizing the maximum hydraulic head and discharge and routing, results in a 3.6-inch diameter orifice (though actual routing may result in a changed orifice size) and Method 2, utilizing average hydraulic head and average discharge, results in a 3.0-inch diameter orifice.

## 2.3.4 Multi-Stage Outlet Design

### 2.3.4.1 Introduction

A combination outlet such as a multiple orifice plate system or multi-stage riser is often used to provide adequate hydraulic outlet controls for the different design requirements (e.g., water quality, channel protection, overbank flood protection, and/or extreme flood protection) for stormwater ponds, stormwater wetlands and detention-only facilities. Separate openings or devices at different elevations are used to control the rate of discharge from a facility during multiple design storms. Figures 2.3.2-3 and 2.3.2-8 are examples of multi-stage combination outlet systems.

A design engineer may be creative to provide the most economical and hydraulically efficient outlet design possible in designing a multi-stage outlet. Many iterative routings are usually required to arrive at a minimum structure size and storage volume that provides proper control. The stage-discharge table or rating curve is a composite of the different outlets that are used for different elevations within the multi-stage riser (see Figure 2.3.2-9).

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### 2.3.4.2 Multi-Stage Outlet Design Procedure

Below are the steps for designing a multi-stage outlet.

- Step 1:** Determine Stormwater Control Volumes. Using the procedures from Sections 2.1 and 2.2, estimate the required storage volumes for water quality treatment ( $WQ_v$ ), channel protection ( $CP_v$ ), and overbank flood control ( $Q_{p50}$ ) and extreme flood control ( $Q_f$ ).
- Step 2:** Develop Stage-Storage Curve. Using the site geometry and topography, develop the stage-storage curve for the facility in order to provide sufficient storage for the control volumes involved in the design.
- Step 3:** Design Water Quality Outlet. Design the water quality extended detention ( $WQ_v$ -ED) orifice using either Method 1 or Method 2 outlined in subsection 2.3.3. If a permanent pool is incorporated into the design of the facility, a portion of the storage volume for water quality will be above the elevation of the permanent pool. The outlet can be protected using either reverse slope pipe, hooded protection device, or another acceptable method (see subsection 2.3.5).
- Step 4:** Design Channel Protection Outlet. Design the stream channel protection extended detention outlet ( $CP_v$ -ED) using either method from subsection 2.3.3. For this design, the storage needed for channel protection will be “stacked” on top of the water quality volume storage elevation determined in Step 3. The total stage-discharge rating curve at this point will include water quality control orifice and the outlet used for stream channel protection. The outlet should be protected in a manner similar to that for the water quality orifice.
- Step 5:** Design Overbank Flood Protection Outlet. The overbank protection volume is added above the water quality and channel protection storage. Establish the  $Q_{p50}$  maximum water surface elevation using the stage-storage curve and subtract the  $CP_v$  elevation to find the 50-year maximum head. Select an outlet type and calculate the initial size and geometry based upon maintaining the predevelopment 50-year peak discharge rate. Develop a stage-discharge curve for the combined set of outlets ( $WQ_v$ ,  $CP_v$  and  $Q_{p50}$ ).

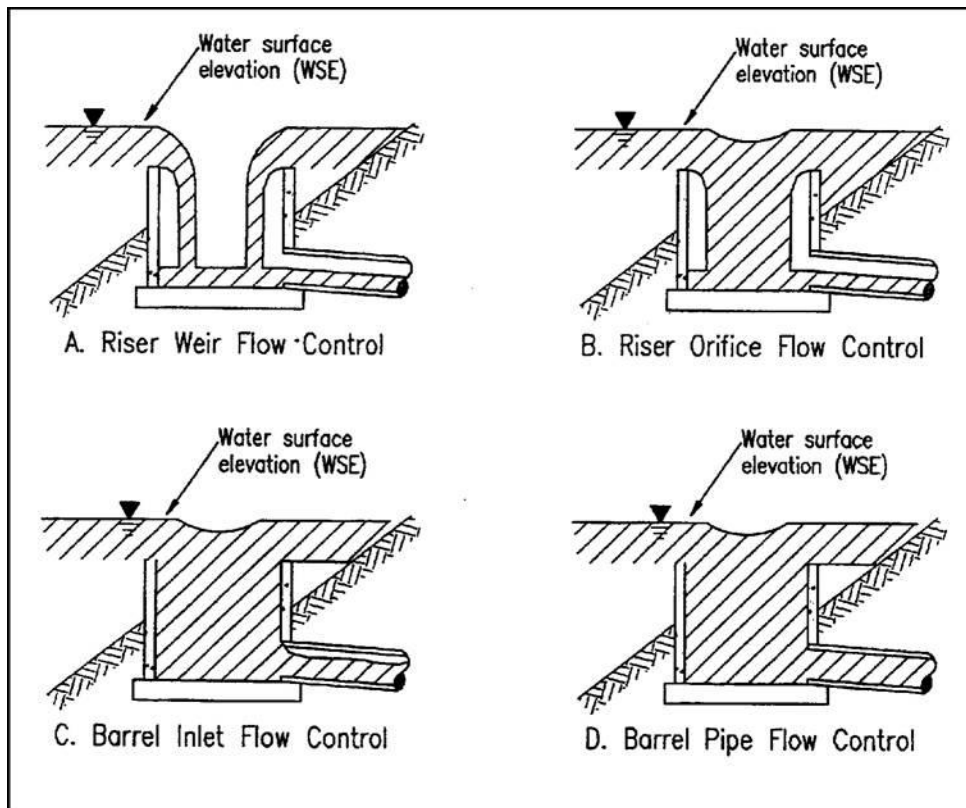
This procedure is repeated for control (peak flow attenuation) of the 100-year storm ( $Q_f$ ), if required.

- Step 6:** Check Performance of the Outlet Structure. Perform a hydraulic analysis of the multistage outlet structure using reservoir routing to ensure that all outlets will function as designed. Several iterations may be required to calibrate and optimize the hydraulics and outlets that are used. Also, the structure should operate without excessive surging, noise, vibration, or vortex action at any stage. This usually requires that the structure have a larger cross-sectional area than the outlet conduit.

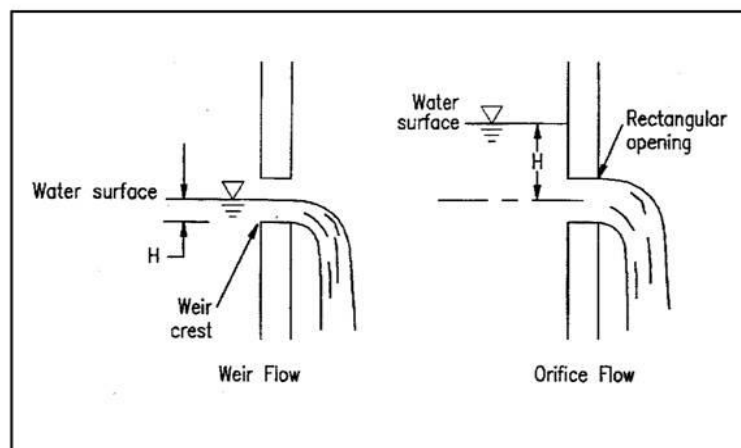
The hydraulic analysis of the design must take into account the hydraulic changes that will occur as depth of storage changes for the different design storms. As shown in Figure 2.3.4-1, as the water passes over the rim of a riser, the riser acts as a weir. However, when the water surface reaches a certain height over the rim of a riser, the riser will begin to act as a submerged orifice. The designer must compute the elevation at which this transition from riser weir flow control to riser orifice flow control takes place for an outlet where this change in hydraulic conditions will change. Also note in Figure 2.3.4-1 that as the elevation of the water increases further, the control can change from barrel inlet flow control to barrel pipe flow control. Figure 2.3.4-2 shows another condition where weir flow can change to orifice flow, which must be taken into account in the hydraulics of the rating curve as different design conditions results in changing water surface elevations.

- Step 7:** Size the Emergency Spillway. It is recommended that all stormwater impoundment structures have a vegetated emergency spillway (see subsection 2.3.6). An emergency spillway provides a degree of safety to prevent overtopping of an embankment if the primary outlet or principal spillway should become clogged, or otherwise inoperative. The 100-year storm should be routed through the outlet devices and emergency spillway to ensure the hydraulics of the system will operate as designed.

- Step 8:** Design Outlet Protection. Design necessary outlet protection and energy dissipation facilities to avoid erosion problems downstream from outlet devices and emergency spillway(s). See Section 4.5, *Energy Dissipation Design*, for more information.
- Step 9:** Perform Buoyancy Calculations. Perform buoyancy calculations for the outlet structure and footing. Flotation will occur when the weight of the structure is less than or equal to the buoyant force exerted by the water.
- Step 10:** Provide Seepage Control. Seepage control should be provided for the outflow pipe or culvert through an embankment. The two most common devices for controlling seepage are (1) filter and drainage diaphragms and (2) anti-seep collars.



**Figure 2.3.4-1 Riser Flow Diagrams**  
(Source: VDCR, 1999)



**Figure 2.3.4-2 Weir and Orifice Flow**  
(Source: VDCR, 1999)

## 2.3.5 Extended Detention Outlet Protection

Small low flow orifices such as those used for extended detention applications can easily clog, thus preventing the structural control from meeting its design purpose(s) and potentially causing adverse impacts. Therefore, extended detention orifices need to be adequately protected from clogging. There are a number of different anti-clogging designs, including:

- The use of a reverse slope pipe attached to a riser for a stormwater pond or wetland with a permanent pool (see Figure 2.3.5-1). The inlet is submerged 1 foot below the elevation of the permanent pool to prevent floatables from clogging the pipe and to avoid discharging warmer water at the surface of the pond.
- The use of a hooded outlet for a stormwater pond or wetland with a permanent pool (see Figures 2.3.5-2 and 2.3.5-3).
- Internal orifice protection through the use of an over-perforated vertical stand pipe with 1/2-inch orifices or slots that are protected by wire-cloth and a stone filtering jacket (see Figure 2.3.5-4).
- Internal orifice protection through the use of an adjustable gate valves can to achieve an equivalent orifice diameter.

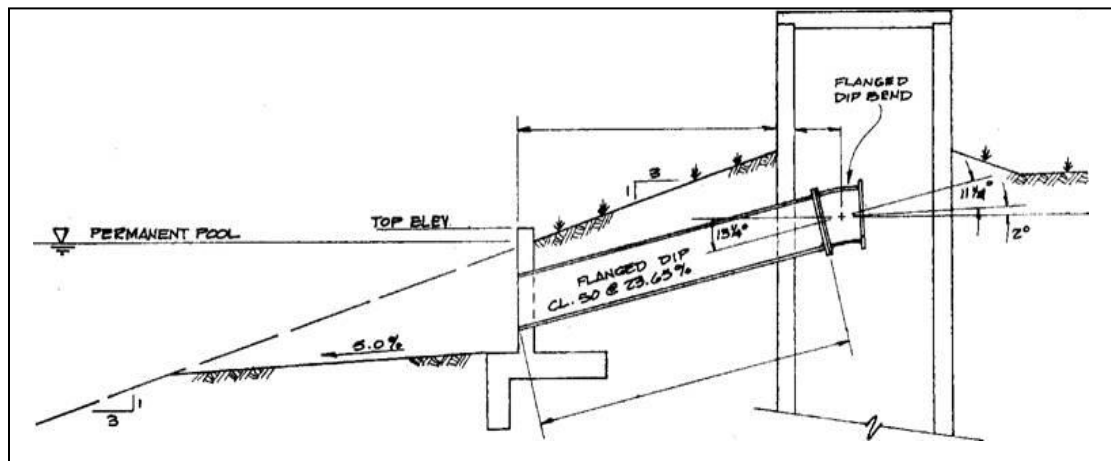


Figure 2.3.5-1 Reverse Slope Pipe Outlet

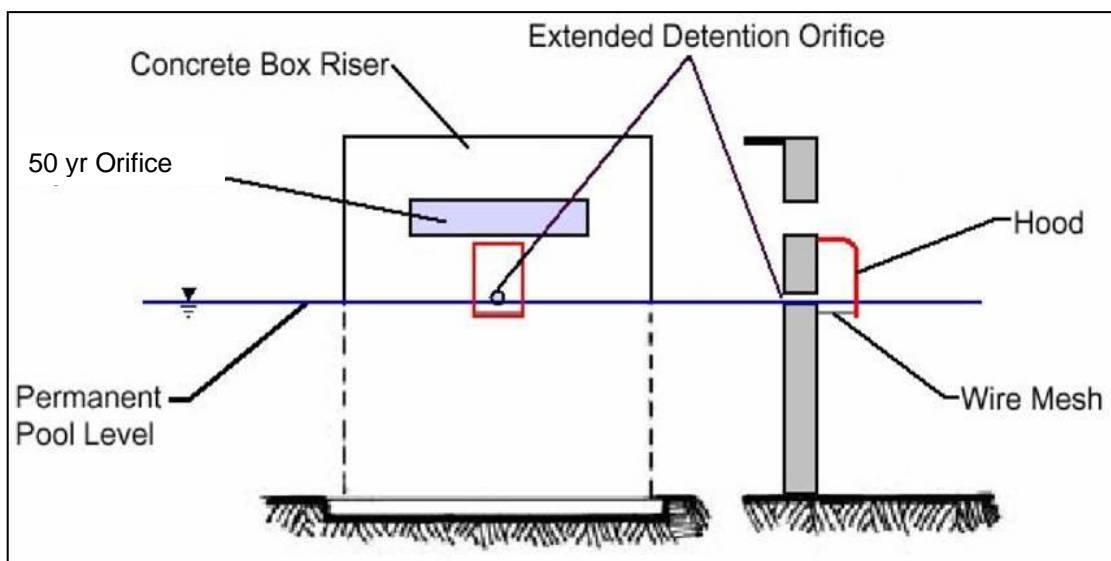
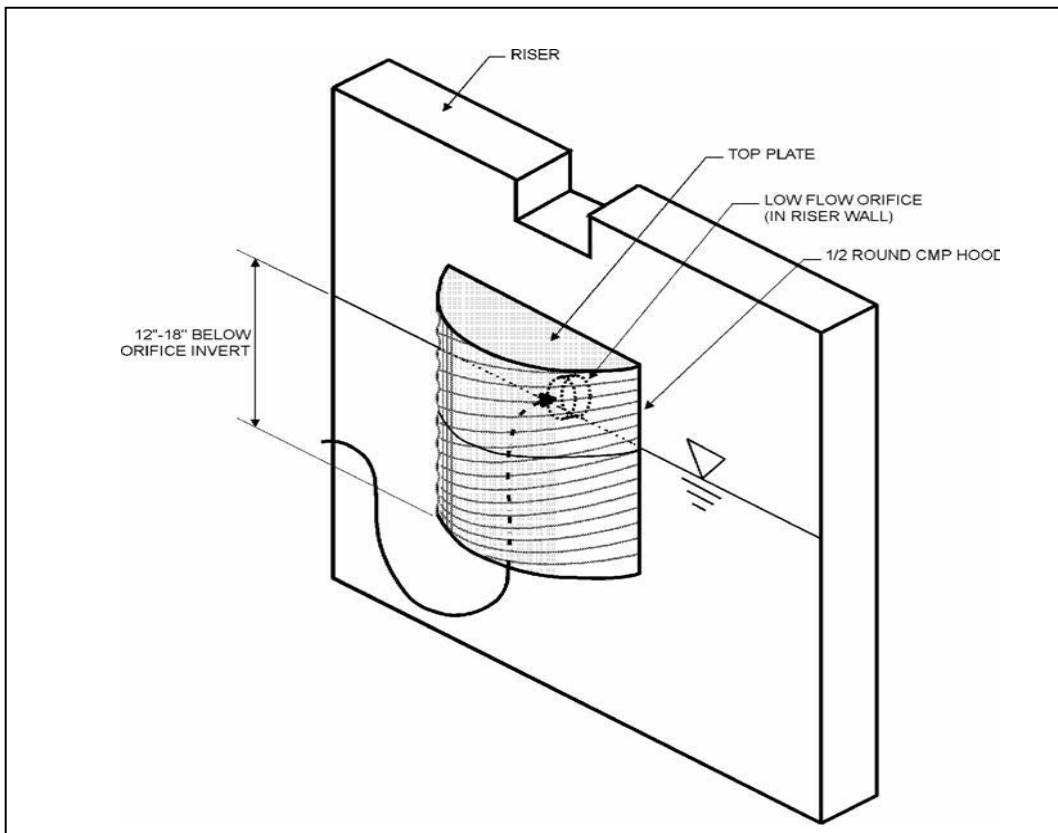
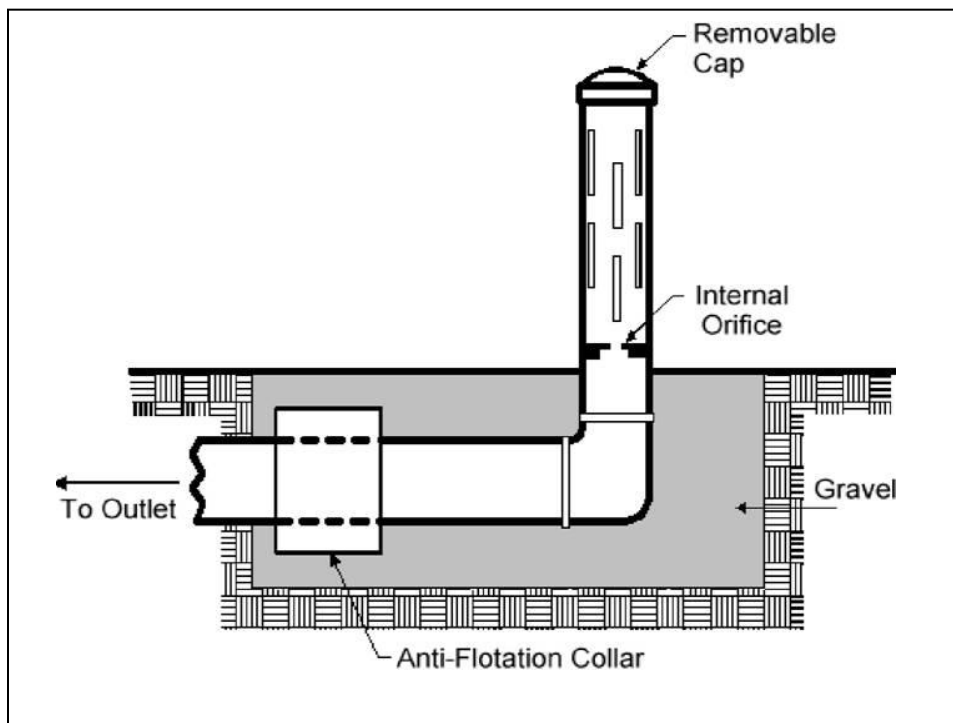


Figure 2.3.5-2 Hooded Outlet



**Figure 2.3.5-3 Half-Round CMP Orifice Hood**



**Figure 2.3.5-4 Internal Control for Orifice Protection**



## 2.3.6 Trash Racks and Safety Grates

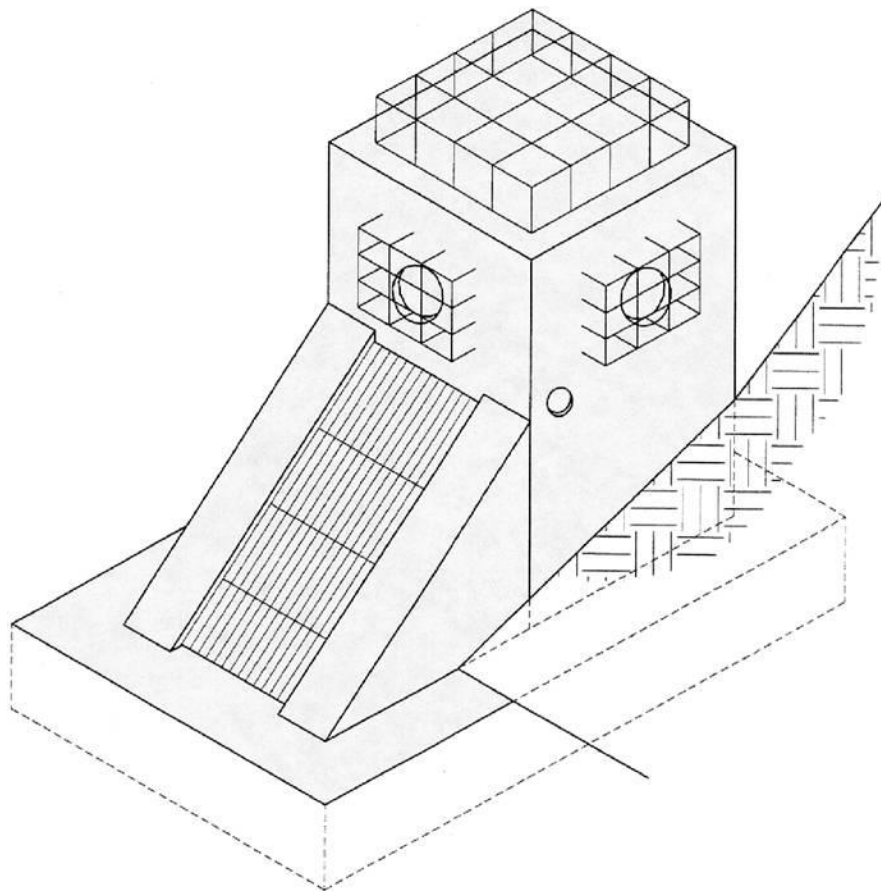
### 2.3.6.1 Introduction

The susceptibility of larger inlets to clogging by debris and trash needs to be considered when estimating their hydraulic capacities. In most instances trash racks will be needed. Trash racks and safety grates are a critical element of outlet structure design and serve several important functions:

- Keeping debris away from the entrance to the outlet works where they will not clog the critical portions of the structure
- Capturing debris in such a way that relatively easy removal is possible
- Ensuring that people and large animals are kept out of confined conveyance and outlet areas
- Providing a safety system that prevents anyone from being drawn into the outlet and allows them to climb to safety

When designed properly, trash racks serve these purposes without interfering significantly with the hydraulic capacity of the outlet (or inlet in the case of conveyance structures) (ASCE, 1985; Allred-Coonrod, 1991). The location and size of the trash rack depends on a number of factors, including head losses through the rack, structural convenience, safety and size of outlet. Well-designed trash racks can also have an aesthetically pleasing appearance.

An example of trash racks used on a riser outlet structure is shown in Figure 2.3.6-1. Additional trash rack design can be found in Appendix C. The inclined vertical bar rack is most effective for lower stage outlets. Debris will ride up the trash rack as water levels rise. This design also allows for removal of accumulated debris with a rake while standing on top of the structure.



**Figure 2.3.6-1 Example of Various Trash Racks Used on a Riser Outlet Structure**

(Source: VDCR, 1999)

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### 2.3.6.2 Trash Rack Design

Trash racks must be large enough such that partial plugging will not adversely restrict flows reaching the control outlet. There are no universal guidelines for the design of trash racks to protect detention basin outlets, although a commonly used "rule-of-thumb" is to have the trash rack area at least ten times larger than the control outlet orifice.

The surface area of all trash racks should be maximized and the trash racks should be located a suitable distance from the protected outlet to avoid interference with the hydraulic capacity of the outlet. The spacing of trash rack bars must be proportioned to the size of the smallest outlet protected. However, where a small orifice is provided, a separate trash rack for that outlet should be used, so that a simpler, sturdier trash rack with more widely spaced members can be used for the other outlets. Spacing of the rack bars should be wide enough to avoid interference, but close enough to provide the level of clogging protection required.

To facilitate removal of accumulated debris and sediment from around the outlet structure, the racks should have hinged connections. If the rack is bolted or set in concrete it will preclude removal of accumulated material and will eventually adversely affect the outlet hydraulics.

Since sediment will tend to accumulate around the lowest stage outlet, the inside of the outlet structure for a dry basin should be depressed below the ground level to minimize clogging due to sedimentation. Depressing the outlet bottom to a depth below the ground surface at least equal to the diameter of the outlet is recommended.

Trash racks at entrances to pipes and conduits should be sloped at about 3H:1 V to 5H:1 V to allow trash to slide up the rack with flow pressure and rising water level—the slower the approach flow, the flatter the angle. Rack opening rules-of-thumb are found in literature. Figure 2.3.6-2 gives opening estimates based on outlet diameter (UDFCD, 1992). Judgment should be used in that an area with higher debris (e.g., a wooded area) may require more opening space.

The bar opening space for small pipes should be less than the pipe diameter. For larger diameter pipes, openings should be 6 inches or less. Collapsible racks have been used in some places if clogging becomes excessive or a person becomes pinned to the rack.

Alternately, debris for culvert openings can be caught upstream from the opening by using pipes placed in the ground or a chain safety net (USBR, 1978; UDFCD, 1992). Racks can be hinged on top to allow for easy opening and cleaning.

The control for the outlet should not shift to the grate, nor should the grate cause the headwater to rise above planned levels. Therefore head losses through the grate should be calculated. A number of empirical loss equations exist though many have difficult to estimate variables. Two will be given to allow for comparison.

Metcalf & Eddy (1972) give the following equation (based on German experiments) for losses. Grate openings should be calculated assuming a certain percentage blockage as a worst case to determine losses and upstream head. Often 40 to 50% is chosen as a working assumption.

$$H_g = K_{g1}(w/x)^{4/3}(V_u^2/2g) \sin \theta_g \quad (2.3.12)$$

Where:  $H_g$  = head loss through grate (ft)

$K_{g1}$  = bar shape factor:

2.42 - sharp edged rectangular

1.83 - rectangular bars with semicircular upstream faces

1.79 - circular bars

1.67 - rectangular bars with semicircular up- and downstream faces

$w$  = maximum cross-sectional bar width facing the flow (in)

$x$  = minimum clear spacing between bars (in)

$V_u$  = approach velocity (ft/s)

$\theta_g$  = angle of the grate with respect to the horizontal (degrees)

The Corps of Engineers (HDC, 1988) has developed curves for trash racks based on similar and additional tests. These curves are for vertical racks but presumably they can be adjusted, in a manner similar to the previous equation, through multiplication by the sine of the angle of the grate with respect to the horizontal.

$$H_g = \frac{K_{g2} V_u^2}{2g} \quad (2.3.13)$$

Where  $K_{g2}$  is defined from a series of fit curves as:

- sharp edged rectangular (length/thickness = 10)

$$K_{g2} = 0.00158 - 0.03217 A_r + 7.1786 A_r^2$$

- sharp edged rectangular (length/thickness = 5)

$$K_{g2} = -0.00731 + 0.069453 A_r + 7.0856 A_r^2$$

- round edged rectangular (length/thickness = 10.9)

$$K_{g2} = -0.00101 + 0.02520 A_r + 6.0000 A_r^2$$

- circular cross section

$$K_{g2} = 0.00866 + 0.13589 A_r + 6.0357 A_r^2$$

and  $A_r$  is the ratio of the area of the bars to the area of the grate section.

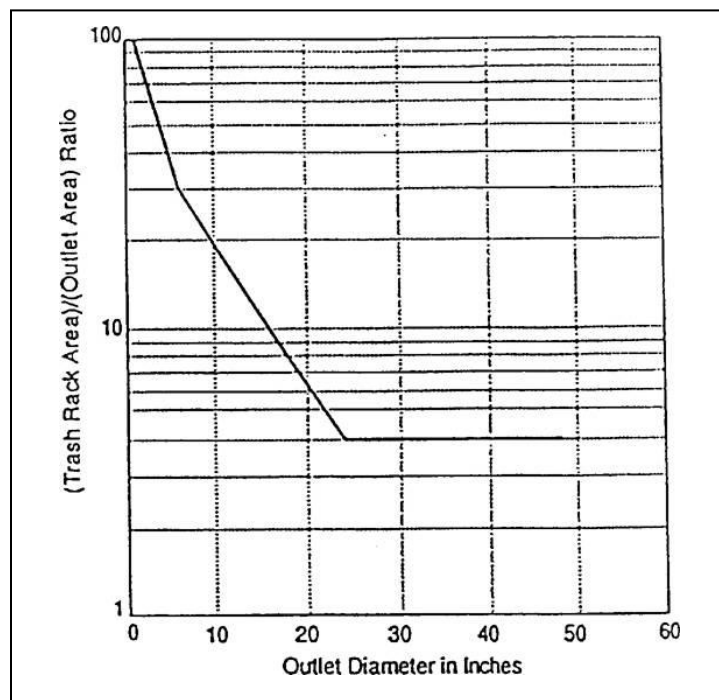


Figure 2.3.6-2 Minimum Rack Size vs. Outlet Diameter

(Source: UDCFD, 1992)

## 2.3.7 Secondary Outlets

### 2.3.7.1 Introduction

The purpose of a secondary outlet (emergency spillway) is to provide a controlled overflow for flows in excess of the maximum design storm for a storage facility. Figure 2.3.7-1 shows an example of an emergency spillway.

In many cases, on-site stormwater storage facilities do not warrant elaborate studies to determine spillway capacity. While the risk of damage due to failure is a real one, it normally does not approach the catastrophic risk involved in the overtopping or breaching of a major reservoir.

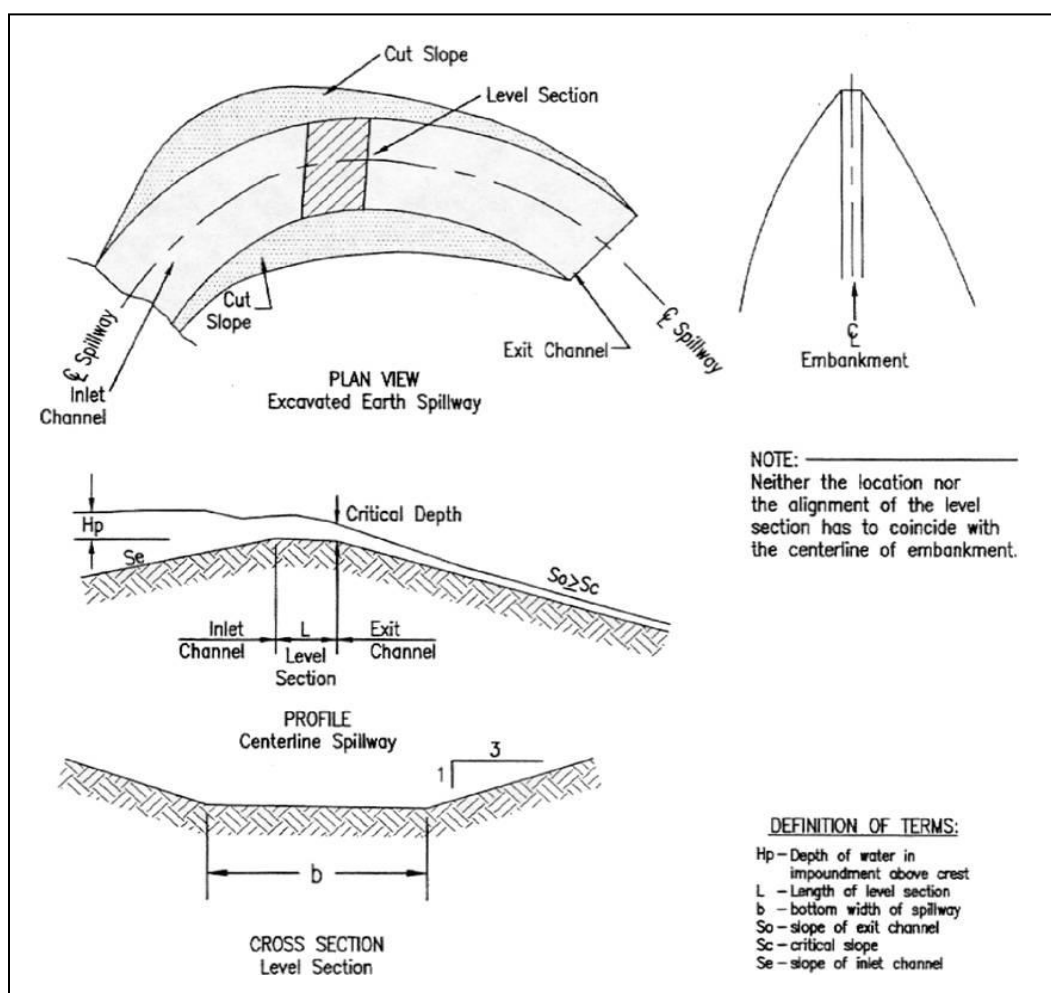
By contrast, regional facilities with homes immediately downstream could pose a significant hazard if failure were to occur, in which case emergency spillway considerations are a major design factor.

### 2.3.7.2 Emergency Spillway Design

Emergency spillway designs are open channels, usually trapezoidal in cross section, and consist of an inlet channel, a control section, and an exit channel (see Figure 2.3.7-1). The emergency spillway is proportioned to pass flows in excess of the design flood (the 100-year flood or greater) without allowing excessive velocities and without overtopping of the embankment. Flow in the emergency spillway is open channel flow (see Section 4.4, *Open Channel Design*, for more information). Normally, it is assumed that critical depth occurs at the control section.

NRCS (SCS) manuals provide guidance for the selection of emergency spillway characteristics for different soil conditions and different types of vegetation. The selection of degree of retardance for a given spillway depends on the vegetation. Knowing the retardance factor and the estimated discharge rate, the emergency spillway bottom width can be determined. For erosion protection during the first year, assume minimum retardance. Both the inlet and exit channels should have a straight alignment and grade. Spillway side slopes should be no steeper than 3:1 horizontal to vertical.

The most common type of emergency spillway used is a broad-crested overflow weir cut through original ground next to the embankment. The transverse cross section of the weir cut is typically trapezoidal in shape for ease of construction. Such an excavated emergency spillway is illustrated below.



**Figure 2.3.7-1 Emergency Spillway**

(Source: VDCR, 1999)